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Time-Settlement Behavior of Processed Refuse Part II: Swelling Pressure and Permeability in Milled Refuse

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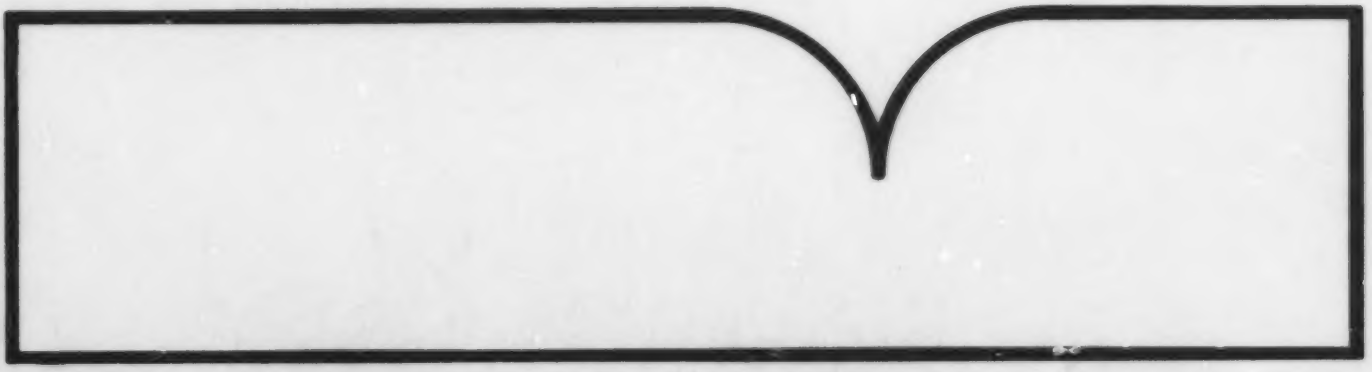

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TIME SETTLEMENT BEHAVIOR OF PROCESSED REFUSE

Part II - Swelling Pressure and Permeability in Milled Refuse

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16. ABSTRACT

This report presents the results of permeability tests and swelling pressure tests on samples of a milled refuse at various compacted dry densities. Permeability tests were done with special attention being given to assure saturation of the sample; for this purpose, the sample was flushed with deaired water and was subjected during testing to back pressures as great as 75 psi. The dependence of engineering permeability on dry density is represented in a semi-logarithmic plot by a bilinear relationship showing a very rapid decrease of permeability with increasing dry density values above approximately 37 lbs/ft^3.

Swelling pressure tests were done by a method designed to approximate a zero deflection condition. It has been observed that potential swelling pressures increase with increasing compacted density; the highest swelling pressure observed being 600 lbs/ft^2 for a dry density of 48.02 lbs/ft^3.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report presents the results of permeability tests and swelling pressure tests on samples of milled refuse at various compacted dry densities. The engineering permeability was investigated with special attention directed toward the degree of saturation.

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ABSTRACT

This report presents the results of permeability tests and swelling pressure tests on samples of a milled refuse at various compacted dry densities. Permeability tests were done with special attention being given to assure saturation of the sample; for this purpose, the sample was flushed with deaired water and was subjected during testing to back pressures as great as 75 psi. The dependence of engineering permeability on dry density is represented in a semi-logarithmic plot by a bilinear relationship showing a very rapid decrease of permeability with increasing dry density for dry density values above approximately 37 lbs/ft^3.

Swelling pressure tests were done by a method designed to approximate a zero deflection condition. It has been observed that potential swelling pressures increase with increasing compacted density; the highest swelling pressure observed being about 600 lbs/ft^2, for a dry density of 48.2 lbs/ft^3.

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ABBREVIATIONS AND SYMBOLS

A_d = cross-sectional area of drainage reservoir, $[L^2]$

A_r = cross-sectional area of supply reservoir, $[L^2]$

A_s = cross-sectional area of sample, $[L^2]$

h_d = head in drainage reservoir, $[L]$

h_d^o = initial head in drainage reservoir, $[L]$

h_d^t = final head in drainage reservoir, $[L]$

h_r = head in supply reservoir, $[L]$

h_r^o = initial head in supply reservoir, $[L]$

h_r^t = final head in supply reservoir, $[L]$

i = hydraulic gradient, dimensionless

k = co-efficient of permeability $[LT^{-1}]$

K = intrinsic permeability, $[L^2]$

a_v = co-efficient of volume compressibility, $[F^{-1}L^2]$

c_v = co-efficient of consolidation, $[L^2T^{-1}]$

e = void ratio, dimensionless

γ_w = unit weight of water, $[F\ L^{-3}]$

η = dynamic viscosity, $[F\ T\ L^{-2}]$

h = hydraulic head, $[L]$

L = length of refuse sample, $[L]$

q = flow rate, $[L^3T^{-1}]$

Q = cumulative discharge, $[L^3]$

t = time, $[T]$

t_o = initial time, $[T]$

t_f = final time, $[T]$

CHAPTER I

INTRODUCTION

One of the recent developments in the technology of refuse disposal is the use of milling, or shredding, processes on waste materials before placement in a sanitary landfill. This method, while still in the experimental stage, appears to be destined for widespread use because of the many advantages it offers over conventional disposal methods.

Among the advantages of the milling process are a reduction in volume, as compared with unprocessed refuse, with consequently smaller settlements; and elimination or reduction of many common landfill nuisances such as rats and blowing paper. An appreciable potential benefit lies in the fact that the comparative homogeneity of the processed refuse reduces the difficulties in treating it as an engineering material. Thus, it can be expected that the potential for beneficial use of the filled sites, after their usefulness in the waste disposal system is ended, will be enhanced.

The objective of this investigation is to examine some of the engineering properties of milled refuse, namely the permeability and potential swelling pressures. The permeability of this material is of significance in its time-settlement behavior, as well as in hydrologic problems associated with sanitary landfills. Swelling pressures are important primarily in the consideration of foundation problems, since foundations on materials with high swelling potential require special consideration in design to protect structures from damage due to volume change.

This report presents the results of a program of permeability tests and swelling pressure tests on samples of milled refuse at various compacted densities. Permeability tests were made with special attention being given to assure saturation of the sample. Swelling pressure tests were made by a method designed to approximate a zero-deflection condition, and so to obtain an estimate of the greatest swelling pressures which might be anticipated.

CHAPTER II

PREVIOUS WORK

While the literature dealing with various aspects of the solid waste disposal problem is voluminous, very little work has been done up to this time in investigating the properties of solid wastes as foundation materials. A general survey of engineering problems associated with sanitary landfills was published by Sowers (1968), who described some of the characteristics of sanitary landfill materials as foundation materials. Some of the difficulties with sanitary landfills as building sites arise from causes other than their mechanical behavior; for instance, sizeable amounts of various gases such as methane are produced, and may cause explosion hazards as well as odor problems.

Permeability of solid waste is lower than might be expected, in view of the nature of the material and the rather low densities usually found. Stone (1968) reported that at a density of 710 pounds per cubic yard a coefficient of permeability of the order of 10^{-4} cm/sec has been observed. At a density of approximately 1100 pounds per cubic yard (equivalent to in situ density) engineering permeability was reported to be too low to measure.

Fagan (1971) found that increasing the density of municipal refuse by compressive load markedly decreased its permeability. Within the density range of 30-65 lbs/cu ft, the logarithm of the coefficient of permeability of the refuse was found to be approximately a linear function of its density. He also noted that permeability of the refuse seemed to vary with time over short time intervals, possibly as a result of incomplete saturation, structural breakdown and/or biological activity.

Hermann, et.al., (1969) investigated permeability earlier, and also reported that the relationship between the logarithm of the coefficient of permeability and the wet density of compressed refuse appeared to be linear. Since these tests were performed without back pressure, the permeability values obtained were probably affected by incomplete saturation.

There is little available information on the bearing capacity of sanitary landfill material. Settlement, however, has received some attention. Sowers (1968) reported that a sanitary landfill will typically settle 10 to 35 percent of its thickness under its own weight. Stone (1968) indicated that settlement is governed by three mechanisms: primary consolidation due to overburden load, secondary compression and creep, and biochemical degradation and decomposition of the material. All three mechanisms are time dependent and are affected by factors such as surface loading, subsurface fill porosity, water content, rainfall, air supply, temperature compactive load , drainage, nutrients for soil bacteria and the location of the ground water table.

Moskowitz (1970) investigated swelling pressures in compressed solid wastes and concluded that for this material the swelling pressures were relatively small. He also recommended that a surcharge sufficient to prevent the material from swelling might be used.

CHAPTER III

MATERIAL TESTED

The primary constituent of sanitary landfill is municipal refuse, which contains a wide variety of materials. Cinders, lumber, unwanted parts of cars, household garbage and construction wastes are a few of the ingredients, yet over half of normal municipal refuse is cellulose. Municipal refuse essentially consists of combustible waste, non-combustible waste, and garbage; the non-combustible waste contains no cellulose, while most of the solids in the other two groups are cellulose. The combustible waste is composed principally of paper, grass and leaves, wood, rags, leather, rubber and synthetics. The paper and rags are nearly completely cellulose and wood is about two-thirds, and the solid components of grass and leaves are about the same as wood. The non-combustible waste consists of metals, glass, stone, ceramics, ashes and soil.

The material which has been studied in the present project is refuse collected from the milling plant at Madison, Wisconsin. The material processed there is unsorted municipal refuse, primarily from city households, which is milled to reduce it to a shredded, relatively homogenious condition. Extensive data on its composition over a period of time have been obtained in The Madison Project (Reinhardt, 1971), and a summary is shown in Table III.1.

The material tested in this project was a sample of 98 kg. collected on May 12, 1971, from the milling plant at Madison. This sample was mixed thoroughly in the laboratory upon arrival and was

Table III.1. Range of composition of solid wastes at Madison, Wisconsin, plant - wet weight basis, (from Reinhardt, 1971).

	MINIMUM (%)	MAXIMUM (%)	AVERAGE (%)
Food wastes	4.4	28.9	15.3
Garden Wastes, primarily grass	0.0	31.1	13.8
Paper products	35.1	53.2	42.4
Plastics, rubber, leather	0.3	3.7	1.8
Textiles	0.1	7.8	1.6
Wood	0.0	2.6	1.1
Metals	5.0	14.5	6.7
Glass and ceramics	4.4	17.6	10.1
Rocks, dirt, ashes etc.	0.6	17.6	7.2

subdivided and placed in air tight plastic bags for storage. It was stored until used at a temperature of 2° C. This series of tests was completed by August 16, 1971.

In the general appearance of the milled refuse from this plant, the most striking characteristics are that it seems to be homogeneous, and its individual components are not easily recognizable. Its appearance is that of torn, somewhat dirty newsprint. The tin cans are normally crumpled or shredded. The glass is disintegrated and has the appearance of angular sand with a maximum particle size of about 1/8 inch. The milled refuse appears to be bulkier than the unprocessed refuse going into the mill. Its bulk is subsequently reduced somewhat by compaction into the truck which carries it to the landfill site.

The predominant particle size of the refuse collected varied from about ½ inch to 2 inches. Since the sample was collected in the spring, the domestic garbage contained an appreciable amount of garden waste, and the sample collected contained an estimated 10% grass and leaves. This garden waste made the visual appearance of the sample very different from that seen in the winter, primarily because of the green coloration. The sample was processed by the Gondard Mill, (Reinhardt, 1971), which on the day of the sample collection was operating with new hammers. As a result, the refuse was shredded relatively finely, and it appeared to be quite homogeneous. A noteworthy exception is that thin plastic bags, pieces of plastic toys, and some cans and pieces of wood were, as usual, not well pulverized. The weight percentage of these materials was small

however by comparison to that of paper. The general appearance of the sample was judged to be quite typical of refuse seen in that season, and there was no indication of any anomalies in its composition.

Reinhardt (1971) indicated that the moisture content of solid waste at this plant varied from 30 to 48 percent, with an average of 37 percent. The average moisture content of the freshly collected refuse sample used for this study was in the range of 36 to 40 percent, as determined by weight loss after drying out at an oven temperature of 105°C. The records of moisture removal from four uniformly mixed representative samples, oven dried for a period of 80 hours at the same temperature, revealed that after 24 hours of drying, about 98 % of the moisture is removed.

CHAPTER IV

SWELLING PRESSURES

Introduction

Due to seasonal variations or under unfavorable climatic conditions some types of foundation materials may undergo expansion with resulting distress to structures. If the potential for swelling is recognized, however, measures can be taken to prevent damage. The motivation for this phase of the study therefore is to evaluate the swelling potential of milled refuse, since this will affect its use as a foundation material.

Moskowitz (1970) studied the swelling characteristics of unmilled, compressed refuse. He described the mechanism of swelling in the waste materials as the absorption of water by the cellulose fibers. He further mentioned that swelling potential in wastes is predominantly controlled by the cellulose content of the wastes. The swelling pressures measured quantitatively in the laboratory might underestimate in situ values. While the confined refuse in the test-cell exerts an upward thrust, a downward frictional resistance is developed around the walls of the mold. Since this friction and adhesion forces between the refuse and the test mold are not taken into account, the observed pressure is less than actually developed in the mold.

An apparatus for measuring swelling pressure in expansive soils was developed by Alpan (1957), who found that a relationship existed between swelling pressures in an expansive soil and confining pressures in a model Using proving rings to measure pressures, he observed that stiffer rings caused greater pressures to be developed and that peak pressures developed in a shorter time interval. Thus, the

compliance of the measuring devices also results in underestimation of swelling pressure potential. The use of a technique in which no deflection is permitted will eliminate both of these sources of error.

Procedures

Sample Preparation

The test specimens were prepared as described in Chapter III from a loose, well mixed sample of milled refuse, with an average water content of 37%. This sample was compacted in the 8 x 8 in. swelling pressure mold with a porous stone at the bottom. Samples at different wet densities, within the range of 35 lbs/ft^3 to 65 lbs/ft^3, were obtained by varying the compacting load, which had a range of 12,000 to 23,700 lbs. The compacting load was applied and measured by means of a 60,000 pound capacity Universal Testing Machine. Prior to testing, the specimen was allowed to rebound freely, constrained only by side-wall friction in the test mold, and the density value used was that attained after the rebound was complete. The amount of rebound was reduced somewhat by holding the press in position at the end of the compression stroke for about 10 minutes to allow stress relaxation in the sample before rebounding.

Test Apparatus

The test apparatus is built around a vertical frame mounted on a flat base, as shown in Figure IV.1. An adjustable screw through the center of the upper horizontal member is fitted with an adapter for the mounting of the proving ring. The frame is constructed entirely of aluminum due to the corrosive nature of the material tested.

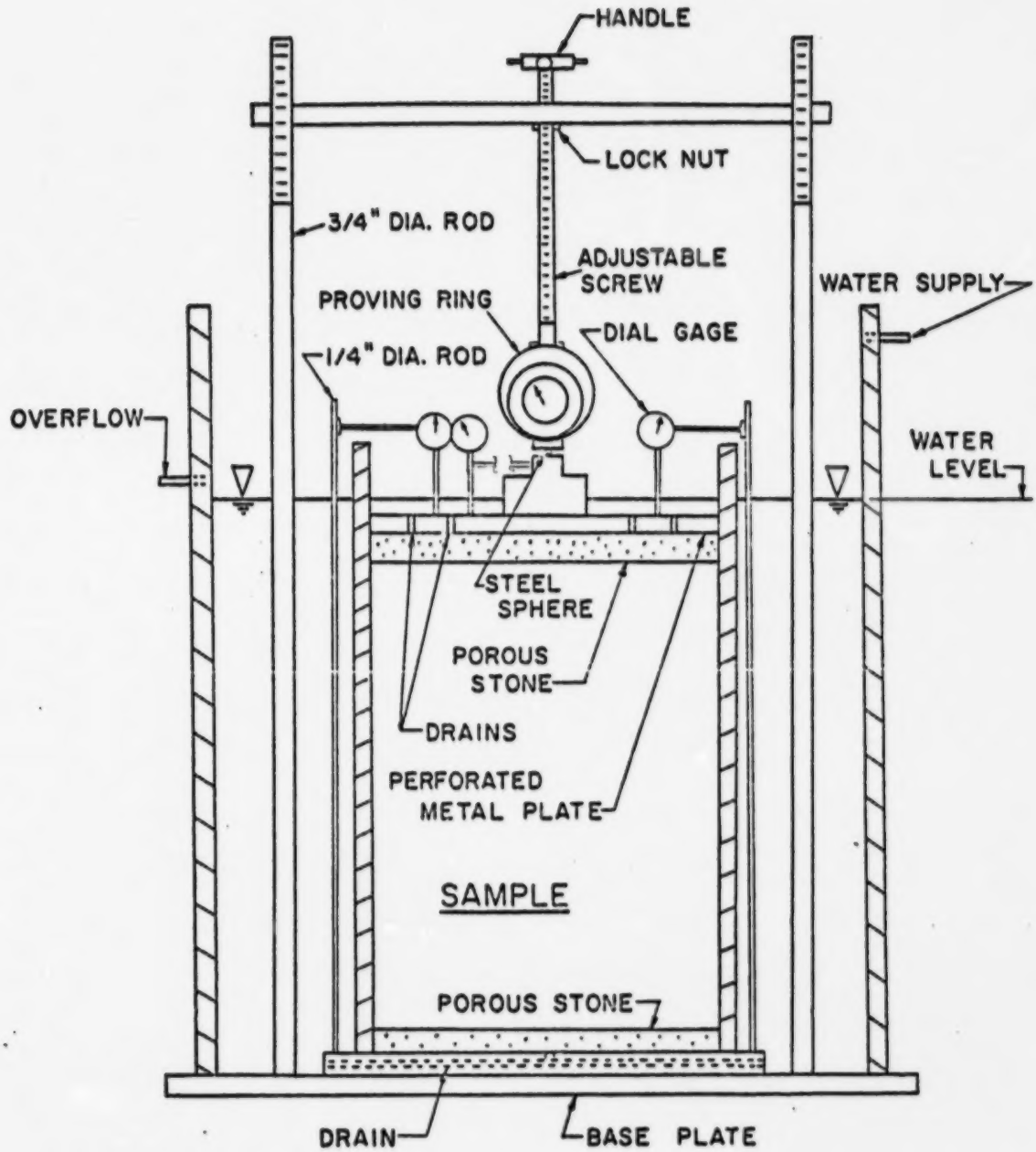

Figure IV.1. Swelling Pressure Test Device (Diagrammatic)

The test cell or sample holder has nominal inside dimensions of 8 inches by 8 inches by 11 inches in height. The cell, constructed by welded ½ inch stainless steel sheet, is bolted to an aluminum plate which is slotted to provide drainage. Porous stones, approximately 3/4 inch thick, were used above and below the sample in the test cell. The stones were prepared in the same manner as those for the permeability tests, as described in Chapter V. A ½ inch thick perforated aluminum plate was used above the top filter stone. An aluminum cylinder 2.5 inches in diameter by 3 inches in height placed at the center of a perforated plate was used as a bearing surface for the proving ring, and to keep the proving ring out of the water in the test cell. The bottom of the proving ring was fitted with 0.5 inch diameter steel cylinder with a spherical bearing surface at the bottom.

Deflections of the upper surface of the sample were measured by three dial gauges with 1/10,000 inch divisions. The gauges were mounted on vertical rods extending upward from three different locations on base plate, so that they were on radii 120 degrees apart. Deflections were measured as the average of the three gauge values.

The swelling pressure apparatus was originally built by Moskowitz (1970), and was modified for this project by providing three deflection gauges in order to provide better control of deflections at the upper surface.

Testing Technique

Since maximum swelling pressures occur when no expansion is permitted, the swelling pressure tests were performed as "zero deflection" tests. In this testing mode, the confining pressure is adjusted by

turning the adjustable vertical screw rod to reverse any small deflection which is observed. Thus, the sample is maintained close to its original height at the points where the deflection gauges are mounted. The average of three gauge readings was used in these tests to maintain a constant volume, although slight uneven swelling resulted in some tilting of the upper plate. Time and deflection data were recorded for each test in this series. For all of the tests total time periods were between about 40 and 115 hours. The time intervals for readings were as in the standard consolidation test (Moskowitz, 1970).

For all tests the zero time point was taken as the time at which the sample was given access to free water. The outer tank was filled with tap water, and access was provided by drains at the top and bottom of the sample. The test cell remained fully submerged in water for the duration of all tests. Tests were run at room temperature and temperature readings in the test tank were taken periodically. Temperatures during these tests remained near 27°C, with a range of ± 1°C.

Test Results

A total of 4 swelling pressure tests were conducted in the course of the laboratory program. Figure IV.2 depicts the swelling pressure behavior with time at different dry densities achieved by different compactive force in the test mold. The maximum swelling pressures were observed to be in the range of 184 to 603 pounds per square foot at different dry densities. In all of the swelling tests the maximum deflections recorded were in the range of about 0.006 to 0.036 inch,

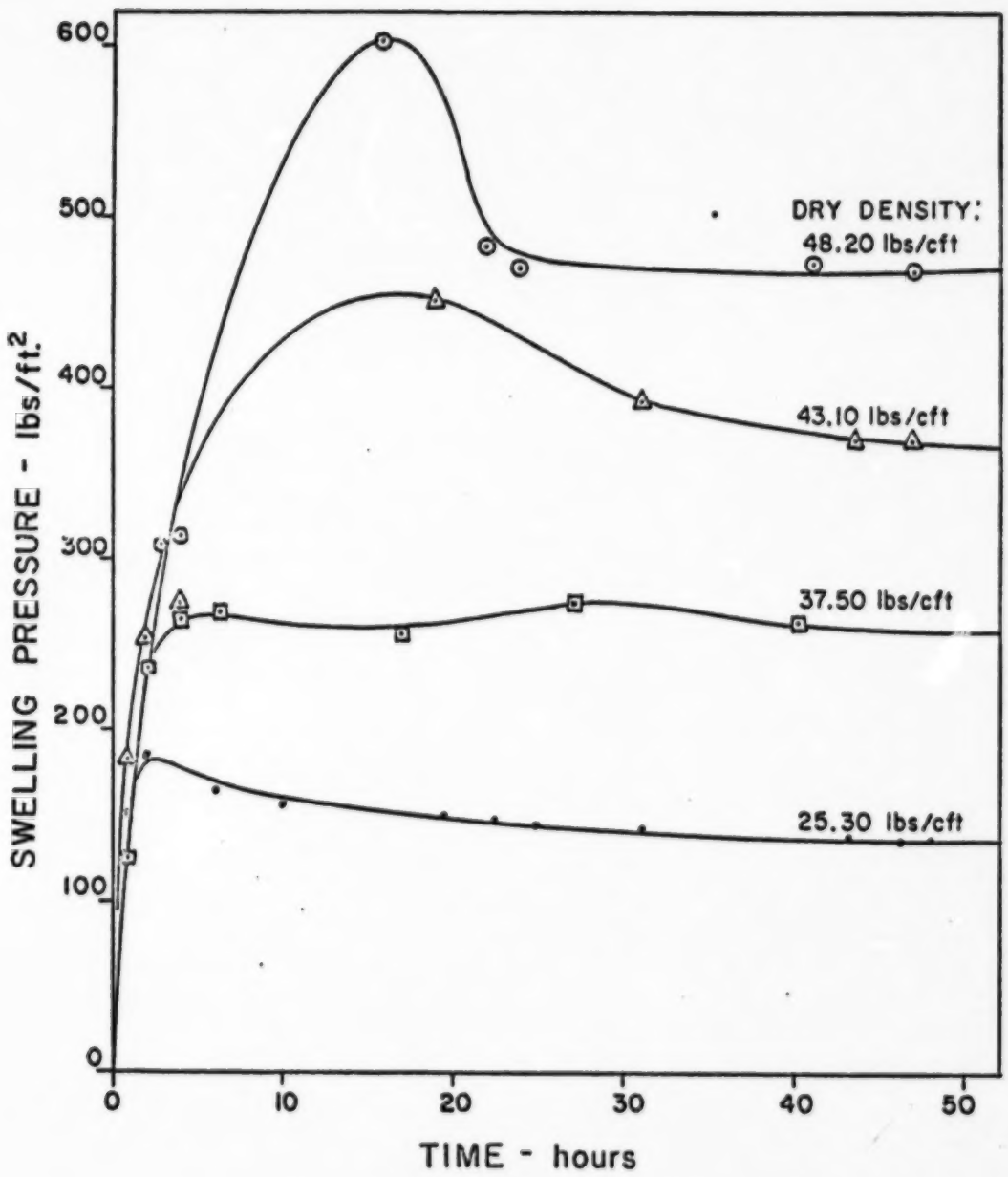

Figure IV.2. Zero Deflection Swelling Pressures of Milled Refuse at Different Compacted Densities

Sample No.	Dry Density lb/ft^3	Max. Compactive Load, lb	Maximum Swelling Deflection 10^{-3} inch	Maximum Swelling Pressure, lb/ft^2
1	25.3	12,000	6.1	184.0
2	37.5	16,200	8.2	268.2
3	48.2	23,700	36.3	603.0
4	43.1	18,800	11.2	450.0

Table IV.1. Summary of Swelling Pressure Tests.

with the greatest deflections occuring in the samples showing the highest swelling pressures. These results are summarized in Table IV.1.

Discussion

The curves of Figure IV.2 show a systematic increase of swelling pressures with compacted dry density, or with the compaction pressure. The curves also indicate that the peak pressure is reached in a relatively short time after access to water is provided, and the pressure relaxes to a steady residual value after a few hours. The time required to reach peak swelling pressure also increases with increasing dry density, probably reflecting lag in water movement due to lower permeability.

The testing method used in this study is termed a "zero deflection" test, although a true zero deflection condition is not realized. A small deflection is actually measured, and the pressure recorded is that required to reverse it. Thus, it is not possible to say with confidence whether this method underestimates or overestimates potential swelling pressure, although it appears to be comparable to a very stiff passive measuring system.

The greatest swelling pressure measured in these tests was 603 lb/ft^2, for a sample with a compacted dry density of 48.2 lb/ft^3. This pressure is equivalent to that produced by a layer of roughly 5 feet of soil. These results contrast to some degree with those of Moskowitz (1970), whose tests on compressed refuse indicated that swelling pressures were quite small; however, the materials tested are not comparable.

CHAPTER V

PERMEABILITY

Introduction

Permeability is a measure of the ease with which a fluid, in this application, water, will flow through a solid possessing continuous, interconnected voids. The notable characteristic that has been observed in such flow is that, within a wide range of flow rates, the movement of water through sands, clays, and even compacted refuse follows the same law first formulated by H. Darcy in 1856 on an empirical basis. The only significant difference is in the magnitude of the coefficient of permeability.

The coefficient of permeability k is defined as the constant of proportionality between discharge velocity and hydraulic gradient in Darcy's law of flow in porous media within the laminar flow range:

$$v=-ki \quad \ldots\ldots(V.1.)$$

where v is the discharge velocity, i is the hydraulic gradient and k is the coefficient of permeability. The hydraulic gradient for the simplest case is given by

$$i = \frac{h}{L} \quad \ldots\ldots(V.2)$$

where h is the hydraulic head difference which provides the driving force and L is the net length of the flow path.

The permeability of a medium is quite important in the consolidation process, the coefficient of permeability being a factor in the coefficient of consolidation,

c_v:

$$c_v = \frac{k(1+e)}{a_v \gamma_w} , \quad \ldots\ldots(V.3)$$

in which a_v is the coefficient of volume compresibility, e is the void ratio of the medium, and γ_w is the unit weight of the permeating fluid.

The coefficient of permeability used in eq. V.1 and throughout this study is the engineering permeability, which is a function of various properties of the solid-fluid system. The "intrinsic permeability", in contrast, is a function only of the porous medium and is independent of the nature of the permeating fluid. The engineering permeability, k, is related to the intrinsic permeability, K, as:

$$k = \frac{\gamma_w}{\eta} K \quad \ldots\ldots (V.4)$$

where γ_w is the unit weight and η is the viscosity of the permeating fluid. The values of viscosity (η) and unit weight of water (γ_w) vary somewhat with temperature, but temperature variations under normal laboratory conditions are small enough that η and γ_w are constant for all practical purposes. Under field conditions this may not be true.

Test Procedures

The apparatus for the permeability test consists of a cylinder in which the sample is held, and provisions for forcing water through the sample under a known head, with measurement of rate of flow. A diagram of the permeability set up as used in this study is shown in Figure V.1. In the falling head type of permeability test, the flow of water is due to a hydraulic head difference which varies with time. The back pressure which is applied both to the drainage and supply reservoirs serves the purpose of reducing the volume of any gas present in the pore space and of encouraging it to go into solution, thus assuring a condition approximating full saturation of voids.

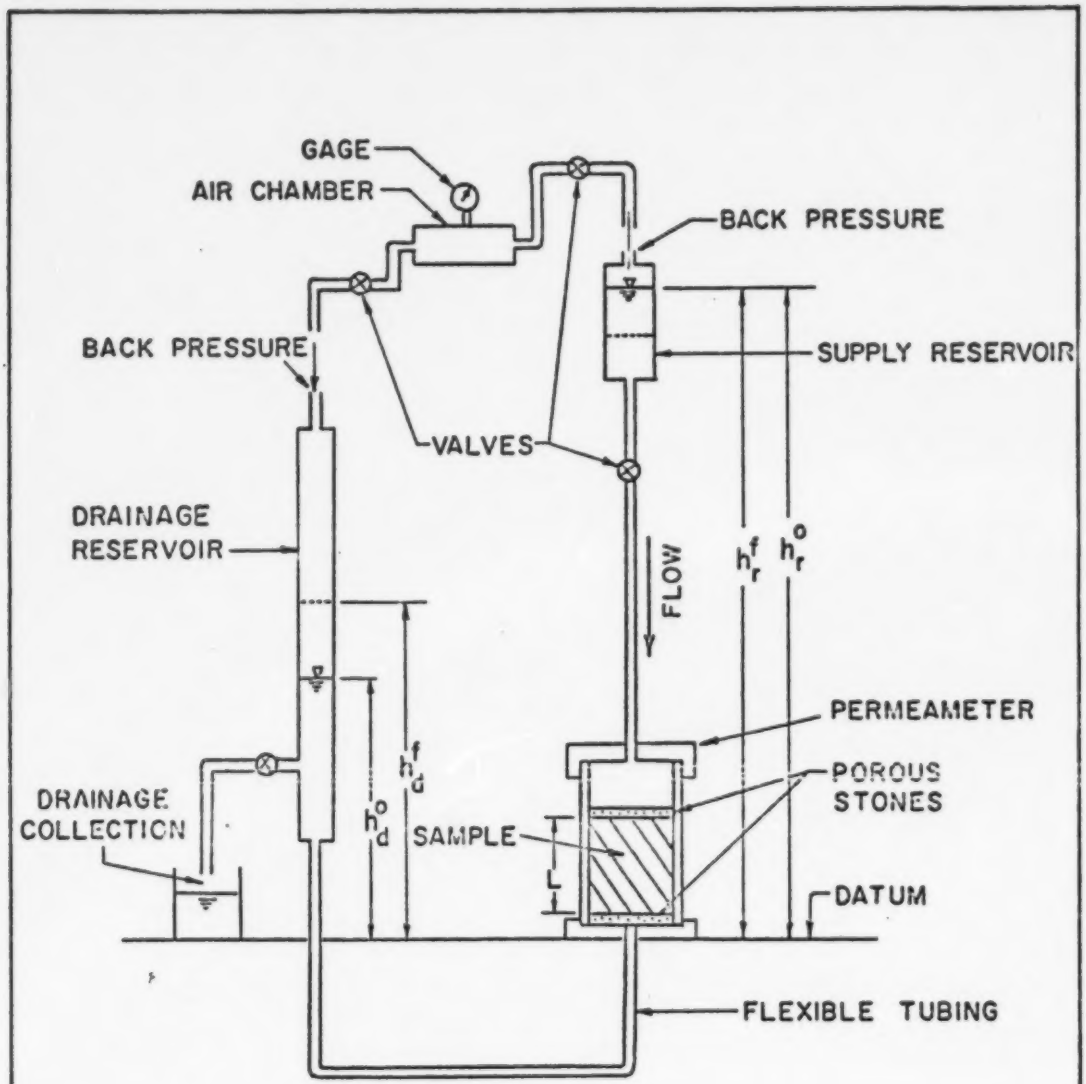

Figure V.1. Falling Head Permeability Test with Back Pressure

The shredded refuse, after removal of any large pieces of plastic or metal, was compacted into the permeameter cylinder under static pressure provided by a hydraulic ram. The permeameter cylinder consists of a two-piece compaction mold with an internal diameter of six inches and a height of nine inches. The refuse was compacted in eight to ten layers, depending on the amount of material required to fill the mold at a particular compaction pressure. Loads applied during compaction procedures ranged from 3,750 pounds for low density samples to 18,000 pounds for the higher density samples. Densities ranged from 17.47 to 63.2 lbs/cu ft. After the sample was compacted, it was allowed to rebound freely in the test cylinder, after which the unit weight was determined and three representative samples were taken for moisture content determination.

The permeameter was sealed with porous stones at both ends. Silicone rubber cement was used as a sealant. The porous stones were prepared in the laboratory using 20-30 Ottawa sand cemented with epoxy resin. It has been found for that particular sand (Anand, 1966) that a sand-resin ratio of 36:1 by weight provides adequate binding with minimal loss of permeability. These porous stones were calibrated for permeability by a falling head test with no refuse sample and without back pressure.

The refuse sample, confined in the mold, was flushed with deaired water under pressure to remove air from the system. At the beginning of the flushing process, leachate flushed from the sample was a dark brownish fluid with a heavy foam layer. In the course of

the test, the drained water showed a light greenish color, with only small amounts of foam or bubbles. Throughout the testing of all samples, the same light greenish color was observed, probably due to the presence of grasses in the refuse samples.

Test Results

The results of the permeability tests are compiled in Table V.1. It is clear from the results that as the density of the refuse increases, the coefficient of permeability decreases, and the plot of permeability versus dry density in Figure V.2 takes a bilinear form on a semi-logarithmic scale. The largest permeability observed was approximately 10^{-2} cm/sec at a dry density of approximately 17 lb/cu ft. With dry densities greater than 38 lb/cu ft, the permeability decreases very rapidly with increasing density. Plots of discharge velocity versus hydraulic gradient for individual samples are given in Figures V.3 through V.8.

The results plotted in Figure V.2 also show the effect of back pressure on the measured permeability values. Since the refuse sample is biochemically active it has a potential to produce gases, though initially in a saturated condition. Evolved gas from the saturated refuse leads to lower values of permeability. Permeability results obtained by Fagan (1970), in which permeability changes with time, are interpreted as effects of incomplete saturation and gas generation in the sample, which was not tested under back pressure.

Discussion

As discussed earlier, permeability is a function of degree of saturation. The problem of insuring essentially complete saturation

Table V.1. Results of Falling Head Permeability Tests

Dry Density (lbs/cu ft) \ Back Pressure (psi)	Compactive force (lbs)	Average Coefficient of Permeability (cm/sec)					
		0	40	50	60	70	75
13.90				9.8×10^{-3}			
17.75		2.8×10^{-3}		9.3×10^{-3}	3.5×10^{-3}		9.9×10^{-3}
23.40		1.3×10^{-3}		4.4×10^{-3}			3.4×10^{-3}
33.35	3,750	0.9×10^{-3}		1.5×10^{-3}			1.6×10^{-3}
40.75	7,550	0.8×10^{-4}	0.9×10^{-4}	1.2×10^{-4}			1.2×10^{-4}
41.80	15,750	2.0×10^{-5}	5.6×10^{-5}	5.3×10^{-5}	5.8×10^{-5}	5.8×10^{-5}	6.4×10^{-5}
45.18	18,000			3.6×10^{-6}			4.1×10^{-6}

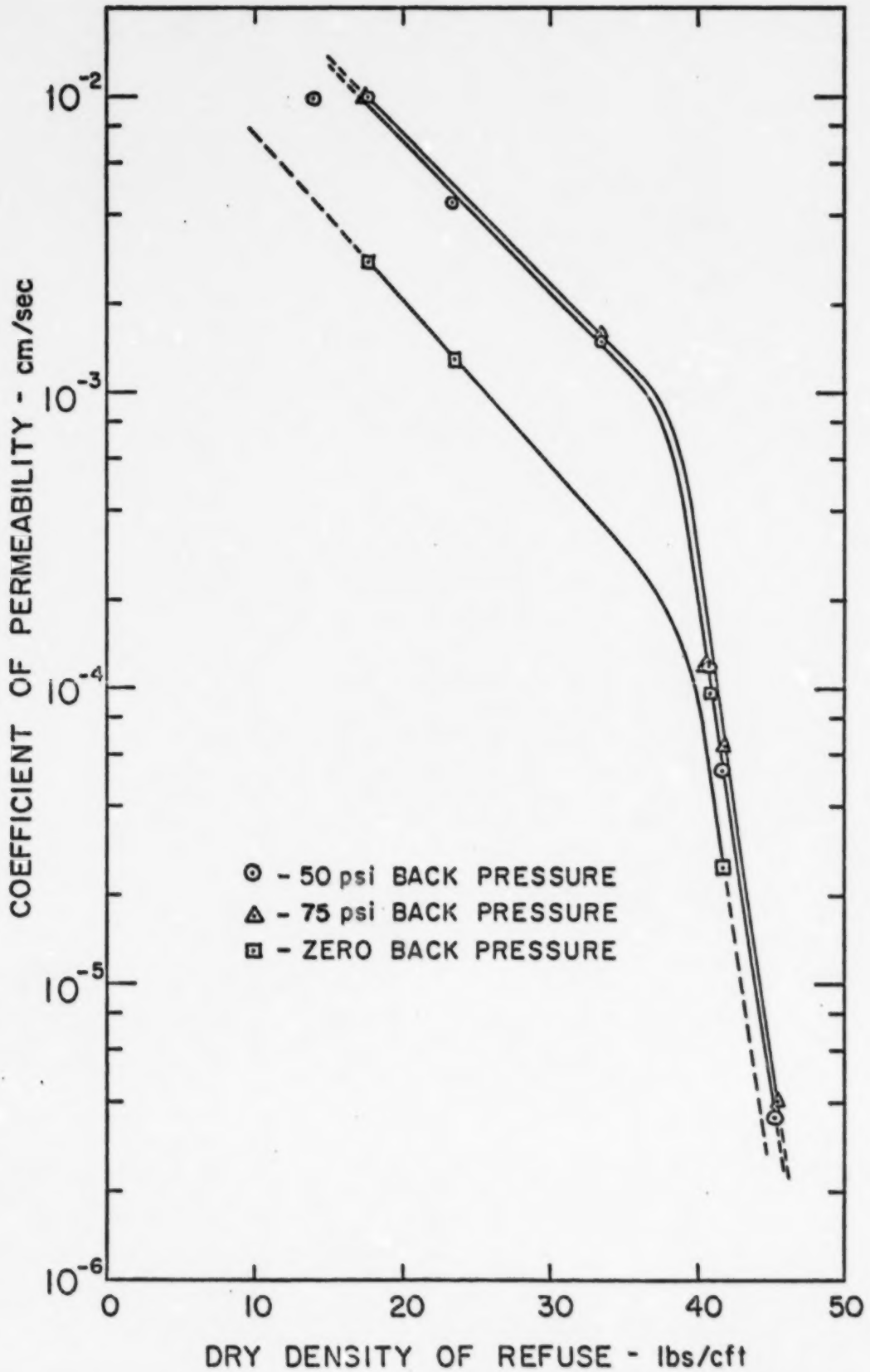

Figure V.2. Permeability versus Dry Density of Milled Refuse

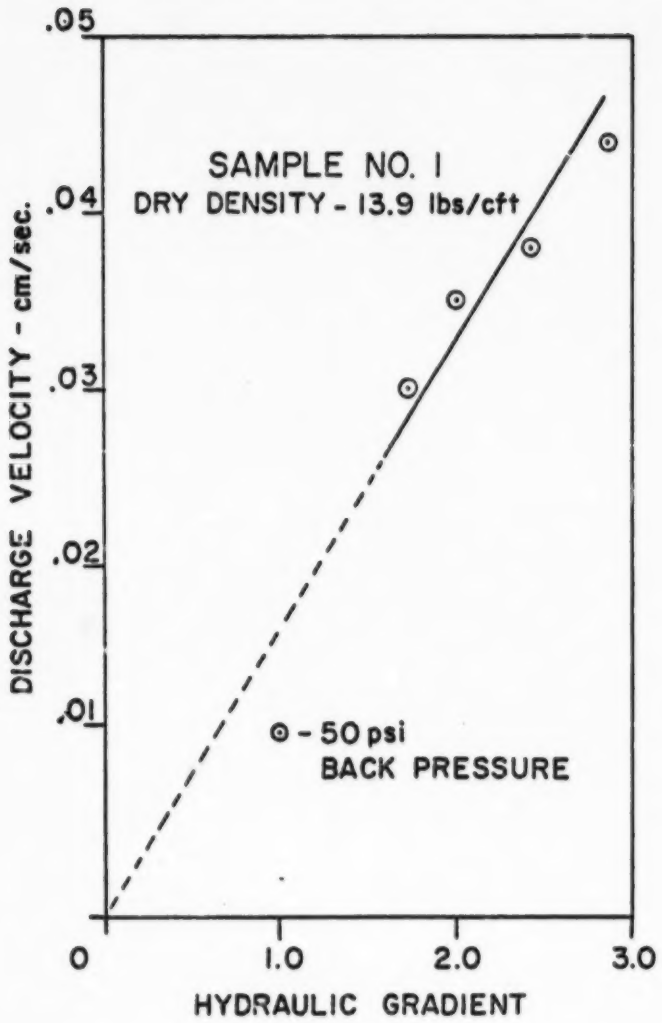

Figure V.3. Discharge Velocity versus Gradient for Sample No.1

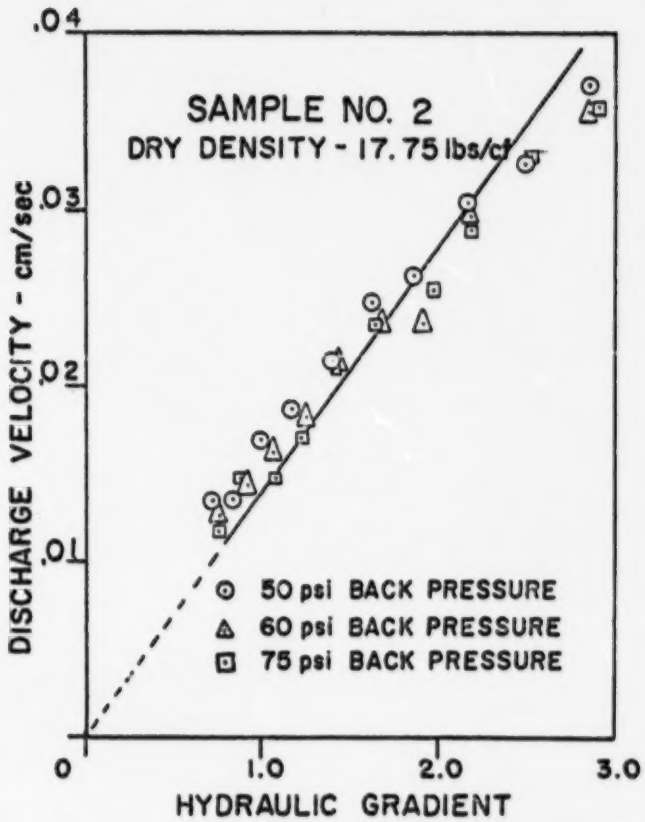

Figure V.4. Discharge Velocity versus Hydraulic Gradient for Sample No.2

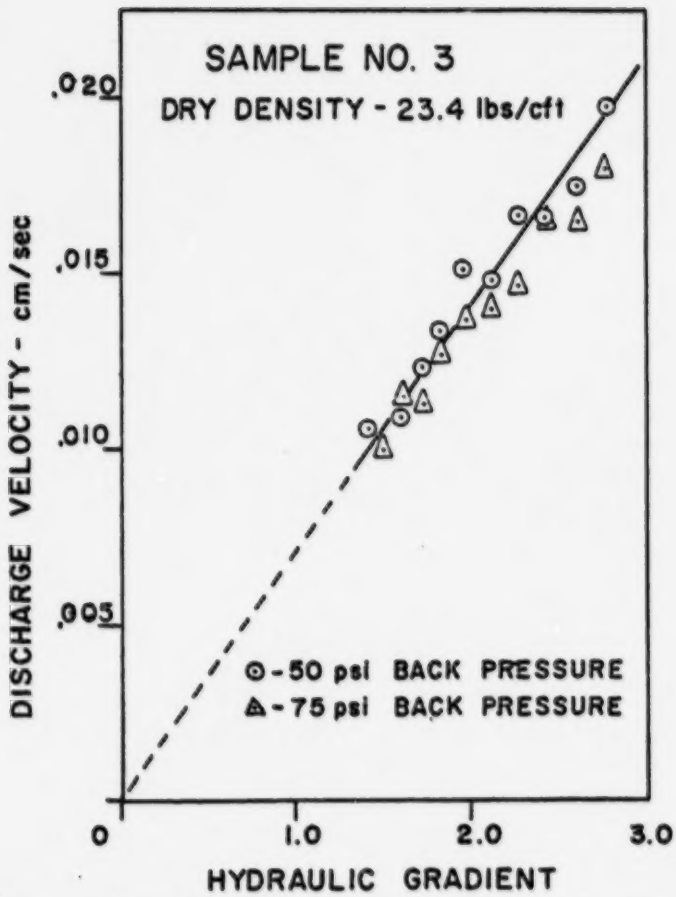

Figure V.5. Discharge Velocity versus Hydraulic Gradient for Sample No.3

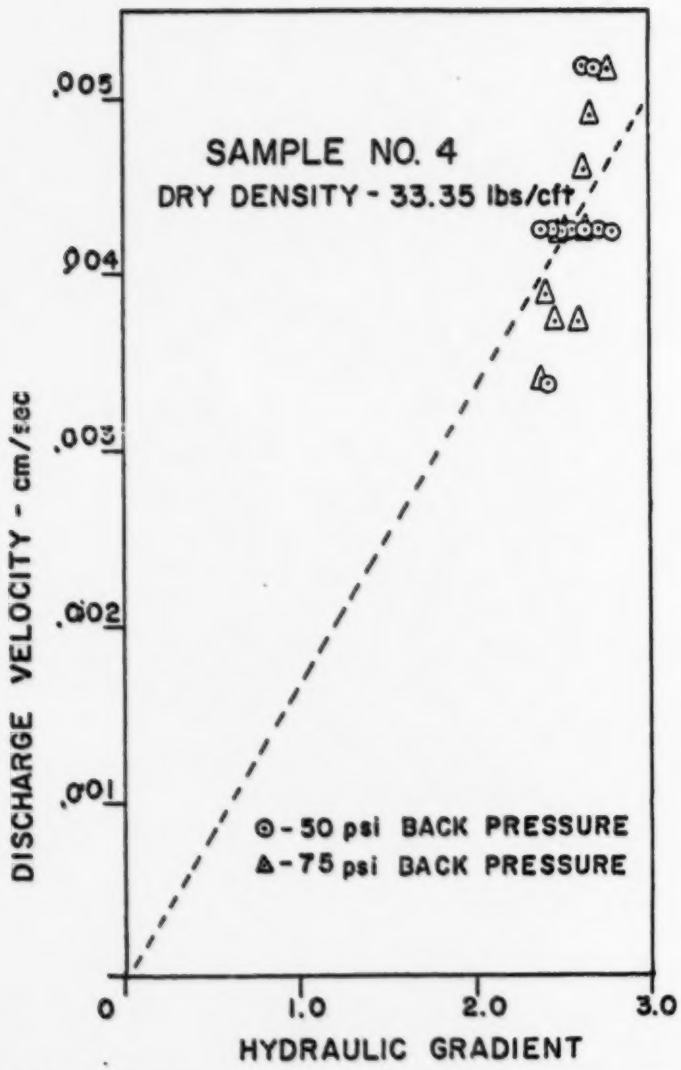

Figure V.6. Discharge Velocity versus Hydraulic Gradient for Sample No.4

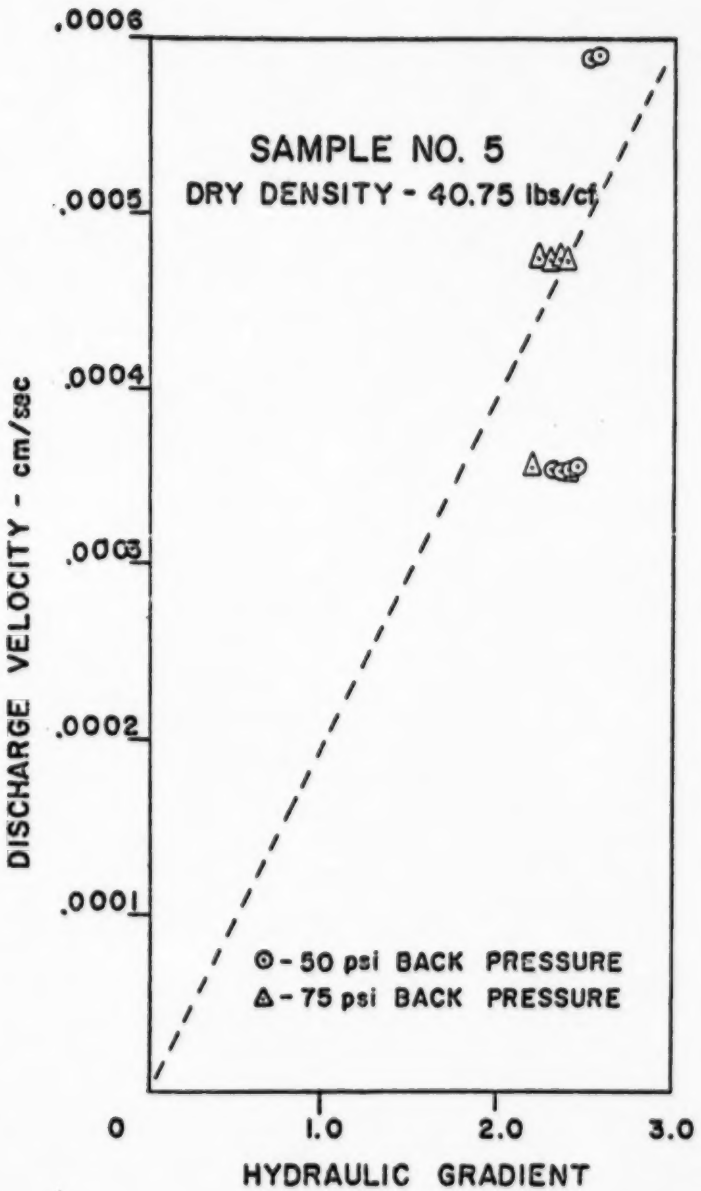

Figure V.7. Discharge Velocity versus Hydraulic Gradient for Sample No.5

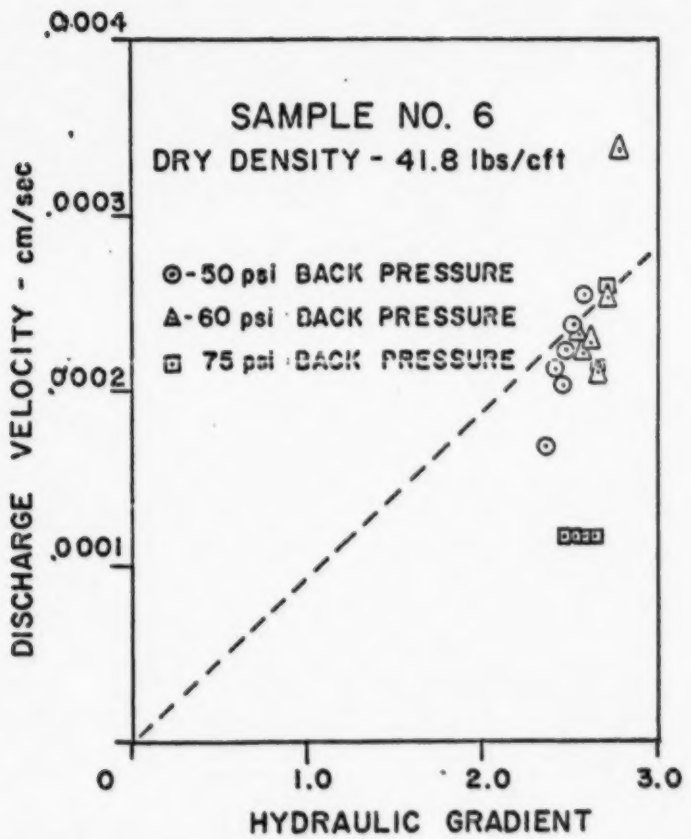

Figure V.8. Discharge Velocity versus Hydraulic Gradient for Sample No.6

has been overcome in this investigation by introducing back pressure in the test system with the result that nearly complete saturation has been attained.

The test results clearly show the expected result that higher densities result in lower permeability values. Unexpectedly, however, the relationship between the dry density of refuse and the logarithm of permeability is distinctly bilinear with a change in slope at an average dry density of 37.5 lbs/cu ft, as seen in Figure V.2. Above that density the permeability decreases very rapidly. Fagan (1971) showed a linear relationship between density and logarithm of permeability within the density range of 30 - 65 lbs/cu ft. Also, he reported in the same investigation, that the permeability varies with time, though he has concluded that this behavior is possibly due to incomplete saturation of the test sample. Hermann, et al. (1969) also reported, in their report on compressed solid wastes, that the relationship between the coefficient of permeability of the refuse and its wet density appeared to be log-linear. Their tests, like Fagan's, were done without back pressure.

Examination of the curves of discharge velocity versus hydraulic gradient in Figures V.6,7, and 8 indicates a pattern differing somewhat from that in Figures V.3,4, and 5: a best-fit straight line through the data points does not pass through the origin, but has an appreciable intercept on the gradient axis. These curves belong to the samples with dry density greater than 37.5 lb/ft^3, the point of change in slope on the bilinear log permeability versus dry density curve. The suggestion of a "threshold gradient" in these samples is interesting, but is not supported by sufficient data to justify the interpretation of the

data on that basis. Accordingly, the permeability has been computed as the average of the discharge velocity:gradient ratios.

CHAPTER VI

SUMMARY AND CONCLUSIONS

Summary

The present investigation consists of a laboratory evaluation of engineering permeability and swelling pressure characteristics of a milled refuse tested at different compacted dry densities. The engineering permeability has been investigated with special attention to the degree of saturation, essentially complete saturation in the sample being obtained by flushing with deaired water and by applying back pressure as great as 75 psi.

Swelling pressure tests on milled refuse samples have been conducted over a range of dry densities from 25.3 lb/ft^3 to 48.2 lb/ft^3. These tests have been performed with lateral confinement in a test cell fitted with an adjustable proving ring, in order to record swelling pressures in approximately a zero-deflection condition.

Conclusions

Subject to limitations in the experimental program imposed by the type of material tested and equipment used, and the manner of sample preparation and control, the following conclusions are drawn:

A. Permeability:

1. For a short term test at complete saturation, the engineering permeability does not change with time.

2. The dependence of engineering permeability on dry density can be represented in a semi-logarithmic plot by a bilinear relationship. This relation shows a very rapid decrease of permeability with increasing

dry density for dry density values above approximately 37 lbs/ft^3.

3. Based on limited data, it can be tentatively concluded that the curves of unsaturated permeability versus dry density in a semi-logarithmic plot are approximately parallel to that at total saturation.

B. Swelling Pressures:

1. The milled refuse has a greater potential swelling pressure than previous tests on unmilled refuse indicate. The highest swelling pressure observed was about 600 lbs/ft^2, for a dry density of 48.2 lbs/ft^3.

2. Potential swelling pressures increase with increasing compacted density.

3. For higher densities of the refuse, longer times are required to reach the peak value of swelling pressure.

4. The swelling pressure, after the peak value is reached, decays with time attaining a constant value which is lower than the peak value.

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APPENDIX A. DERIVATION OF PERMEABILITY EQUATION

As discussed in Chapter V, the permeability tests for this investigation were performed with back pressure in order to assure saturation of the test sample. Because of the physical arrangements made to provide for this, the variables in the test are somewhat different from those in the standard laboratory test methods. An analysis of this test is presented in the following paragraphs.

It is assumed that Darcy's Law applies, that the sample is saturated, that the sample is homogeneous and isotropic with respect to permeability, and that the permeability is unaffected by the back pressure. Referring to Figure V.1, we will use the following notation:

h_d^o = initial head in drainage reservoir; i.e. at $t = t_o$

h_d^f = final head in drainage reservoir; i.e. at $t = t_f$

h_d = head in drainage reservoir at time t

h_r^o = initial head in supply reservoir; i.e. at $t = t_o$

h_r^f = final head in supply reservoir; i.e. at $t = t_f$

h_r = head in supply reservoir at any time t

A_d = cross-sectional area of drainage reservoir

A_r = cross-sectional area of supply reservoir

A_s = cross-sectional area of sample

L = length of sample

Darcy's Law can be written

$$v = -ki \qquad \text{(A-1)}$$

in which v is the discharge velocity, k the coefficient of permeability, and i the hydraulic gradient. Since the discharge velocity is defined as the ratio of the flow rate q to the cross-sectional area through which flow occurs, and the average hydraulic gradient in the sample is given

by $(h_r - h_d)/L$, Darcy's Law for this case can be written

$$q = k A_s \frac{h_r - h_d}{L} \tag{A-2}$$

The condition of continuity at any time for incompressible flow gives

$$Q = (h_r^o - h_r) A_r = (h_d - h_d^o) A_d \tag{A-3}$$

where Q is the cumulative discharge. Solving for h_d gives

$$h_d = (h_r^o - h_r) \frac{A_r}{A_d} + h_d^o \tag{A-4}$$

The discharge dQ in any time interval dt is

$$dA = qdt = -A_r dh_r \tag{A-5}$$

Hence, from equations (A-2), (A-4), and (A-5), we get

$$-A_r dh_r = \frac{kA_s}{L} \left[h_r - (h_r^o - h_r) \frac{A_r}{A_d} - h_d^o \right] dt$$

$$= \frac{kA_s}{L} \left[h_r \left(1 + \frac{A_r}{A_d} \right) - h_r^o \frac{A_r}{A_d} - h_d^o \right] dt,$$

or,
$$\frac{dh_r}{dt} = \frac{kA_s}{L} \left[h_r \left(\frac{1}{A_r} + \frac{1}{A_d} \right) - \frac{h_r^o}{A_d} - \frac{h_d^o}{A_r} \right]. \tag{A-6}$$

this is an ordinary differential equation of the form

$$y' + ay = b, \tag{A-7}$$

which has the general solution

$$y = \frac{b}{a} + ce^{-ax}. \tag{A-8}$$

Comparing equations, (A-6) and (A-7), we get

$$a = \frac{kA_s}{L} \left(\frac{1}{A_r} + \frac{1}{A_d} \right)$$

$$= \frac{kA_s}{L}\left(\frac{A_r + A_d}{A_r A_d}\right) \tag{A-9}$$

and $$b = \frac{kA_s}{L}\left(\frac{h_r^o}{A_d} + \frac{h_d^o}{A_r}\right). \tag{A-10}$$

Hence, the general solution is

$$h_r = \left(\frac{h_r^o}{A_d} + \frac{h_d^o}{A_r}\right)\left(\frac{A_r A_d}{A_r + A_d}\right) + ce^{-\frac{kA_s}{L}\left(\frac{A_r + A_d}{A_r A_d}\right)t} \tag{A-11}$$

Substituting the initial conditions, at $t = 0$, $h_r = h_r^o$, we have

$$c = h_r^o - \left(\frac{A_r h_r^o + A_d h_d^o}{A_r + A_d}\right). \tag{A-12}$$

Substituting equation (A-12) into (A-11) gives

$$h_r = \left(\frac{h_r^o}{A_d} + \frac{h_d^o}{A_r}\right)\left(\frac{A_r A_d}{A_r + A_d}\right) + \left(h_r^o - \frac{A_r h_r^o - A_d h_d^o}{A_r + A_d}\right)e^{-\frac{kA_s}{L}\left(\frac{A_r + A_d}{A_r A_d}\right)t} \tag{A-13}$$

substituting the final condition, at $t = t_f$, $h_r = h_r^f$, we have

$$h_r^f = \left(\frac{h_r^o}{A_d} + \frac{h_d^o}{A_r}\right)\left(\frac{A_r A_d}{A_r + A_d}\right) + \left(h_r^o - \frac{A_r h_r^o - A_d h_d^o}{A_r + A_d}\right)e^{-\frac{kA_s}{L}\left(\frac{A_r + A_d}{A_r A_d}\right)t_f} \tag{A-14}$$

or,
$$\frac{kA_s}{L}\left(\frac{A_r + A_d}{A_r A_d}\right)t_f = -\ln\left\{\frac{h_r^f + \left(-\frac{h_d^o}{A_r} - \frac{h_r^o}{A_d}\right)\left(\frac{A_r A_d}{A_r + A_d}\right)}{\frac{h_r^o\left(A_r + A_d\right) + A_d h_d^o - A_r h_r^o}{\left(A_r + A_d\right)}}\right\} \tag{A-15}$$

Finally, solving for k gives

$$k = -\frac{L}{A_s t}\left(\frac{A_r A_d}{A_r + A_d}\right) \ln\left\{\frac{(h_r^f - h_d^o)\,A_d + (h_r^f - h_r^o)\,A_r}{(h_r^o - h_d^o)\,A_d}\right\}$$

$$= \frac{A_r\,A_d\,L}{A_s(A_r + A_d)\,(t_f - t_o)} \ln\left\{\frac{(h_r^o - h_d^o)\,A_d}{(h_r^t - h_d^o)\,A_d + (h_r^t - h_r^o)\,A_r}\right\}$$

(A-16)